CONCRETE PAVEMENT STRENGTH AND BEAM FLEXURAL STRENGTH

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ABSTRACT

The complete failure of a concrete pavement can be divided into three stages: from brand new to a crack initiated; from the first crack initiation to full depth and full length; and from one crack to many cracks those lead to end of pavement service life. The validation of "fatigue failure" concept embedded in FAA design specifications since 1970's was based on an assumption: the concrete pavement strength is relatively close to the concrete beam flexural strength following ASTM C78. In past ten years, this assumption has been repeatedly verified by the full scale tests for different pavements under static and slow rolling loads at the Federal Aviation Administration's (FAA's) National Airport Pavement Test Facility (NAPTF). However, different conclusion was obtained recently based on experimental studies in [8] and [9]: the pavement strength is 2.8 times in average higher than the beam flexural strength. Their test procedures were reviewed and data were re-analyzed. It has been found that the test results themselves are reliable, but the analysis procedure leads to an overestimate of pavement strength. Three-stage failure was clearly recorded from the tests. The response at the end of second stage was analyzed using linear-elastic model that is only valid in the first stage. After the data is re-analyzed up to the end of the first stage, the conclusions by the FAA and [8][9] become similar.

FROM TWO-STAGE TO THREE-STAGE FAILURE MODEL

Three Stage Failure is a new concept developed based on the two-stage failure model proposed by Rollings in 1988, [1], Figure 1.

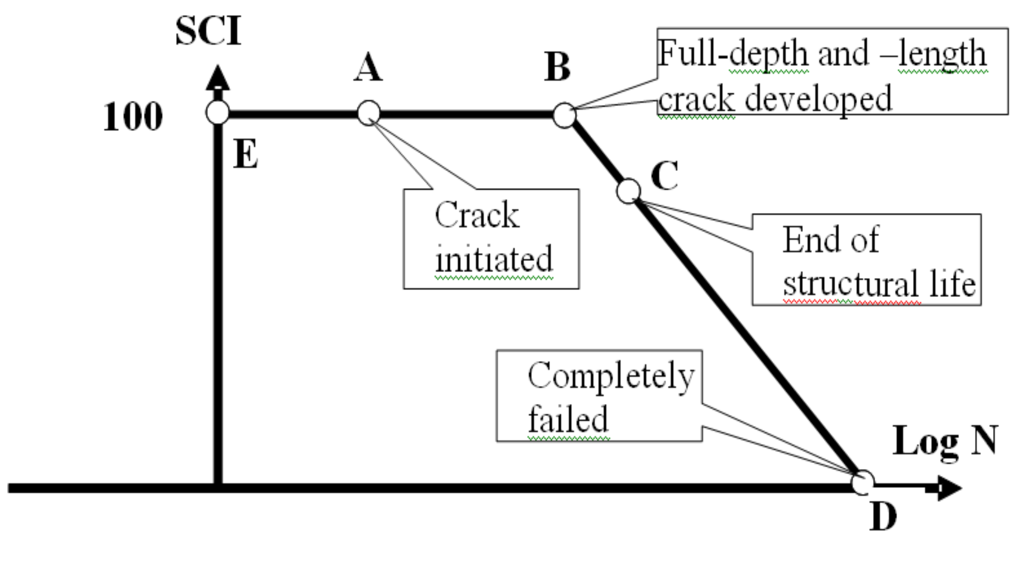


Figure 1 The Failure model for airport pavement design

Pavement structural condition index (SCI) has been defined to describe the pavement current structural condition. Six distresses are defined as "structure" related for concrete pavements that lead to decrease of the SCI: (1) Corner break, (2) Longitudinal, transverse and diagonal crack, (3) Shattered slab or intersecting cracks; (4) Shrinkage cracks; (5) Joint Spalling: (6) Corner

Spalling. For considering effects of above distresses, the index SCI is calculated as the difference between 100 and reduction of SCI due to all observed distresses mentioned above. The detailed discussion of concepts and numerical calculations can be found in [2].

Three key points were originally defined by Rollings [1] in Figure 1 used for design. "E" indicates a new constructed pavement with SCI = 100. "B" is the turning point from SCI = 100 to below 100, indicates that the first full depth and end to end (full length) crack has been developed. "C", ending point of pavement operational life (SCI = 80 for pavements in many large hubs). Apart from above three points, "D", completely failure, SCI = 0, is conceptually equivalent to crushed stones. The x-axis represents the coverage number N (in Log scale) of the maximum stresses in the pavement and it is related to the pavement structural life in design. Therefore, Rollings' model considers two failure stages: from new pavement to the first full-depth and full-length crack developed (E to B in Figure 1); and from one to many cracks leading the end of pavement life (B to C in Figure 1).

Point A is now added into Rollings' model in Figure 1 for indicating the initiation of the crack of concrete pavement, regardless it starts from slab bottom or surface. Now the three-stage failure model has been generated: stage one is from E to A; stage two from A to B and stage three from B to C. Full scale tests conducted at the FAA's NAPTF in past ten years clearly show the three stages: the load pass number was recorded from 1 to more than 2000 to initiate the first crack; and a few thousands of passes led a bottom-up crack from initiation to full depth; and only a fraction of above pass number led to a top-down crack completed; and many thousands of passes were recorded to bring the test pavement from the first crack to end of pavement life – SCI = 80 or lower. Stage by stage seems a better way to understand the failure mechanism of concrete pavement.

DEFINITIONS

Concrete Flexural Strength is a parameter that indicates the capacity of concrete to withstand bending stress. It can be determined following the standard test method ASTM C78 [11]. Though it has been popularly used for concrete pavement design, the test results will vary where there are differences in specimen size, preparation, moisture condition, or where the beam has been molded or sawed.

Strength of a Concrete Pavement is an index indicating the resistance capability (or "quality") of the pavement against crack initiation. The pavement strength can be quantified by the critical stress leading to the initiation of crack in the concrete pavement under a single repetition of load. The "critical stress" is the total stress due to all effects, including load and environmental variations, rather than the stress due to a load only. Up to now, only the load induced stress, rather than the total stress, can be accurately measured. The load induced stress is only a portion of the total stress in concrete pavement and it could be lower or higher than the total stress. However, the stresses employed in all existing models for airport pavement design are "load induced" rather than the "total" stress. This is because the reliability of the total stresses predicted by all mechanistic models still needs to be verified while the technique for measuring the total stress (or total stress related strain) is still being under development.

Fatigue Strength of Concrete Pavement When the critical stress in the pavement is lower than the pavement strength under the same load, it needs multiple repetitions of the load to initiate the crack. The related stress may be defined as fatigue strength of pavement. The higher the stress leads to less of the load repetitions to initiate a crack. Or, there exist no unique fatigue strength for a pavement, rather, the fatigue strength of a pavement is a function of N_I – number of load repetition to lead the crack initiation.

TWO BASIC REQUIREMENTS FOR A VALID "FATIGUE ANALYSIS"

Two basic requirements are worth to be aware of in fatigue analysis. First, the ratio σ/R, rather than σ and/or R, is used as an independent parameter in fatigue analysis. In which, σ is the critical stress and R is the strength in a specimen or in a structure. This requirement has been satisfied by all nine design models for airport pavement reviewed in [3][4], Figure 2. However, seven of the nine models, except the two curves those pass the point: N = 1, $\sigma/R = 1$ in Figure 2, do not satisfy the second basic requirement for fatigue analysis in concept: the flexural strength and the stress should be obtained from the same specimen or structure [5]. In the seven models, MR is obtained from a beam Lab test as a material property, but σ is calculated from pavement as a structural response using different models. They are not obtained from the same structure. That is why none of the seven models (curves) in figure 2 passes, even close to the point (N = 1, $\sigma/R = 1$). Or, the findings by fatigue analysis in many publications might be neither valid nor applicable for the "true fatigue analysis" of concrete pavement, unless the value MR as a material property is close to the concrete pavement strength as a structural response. Therefore, how different the values of beam flexural strength and the concrete pavement strength become an important issue in developing a design specification.

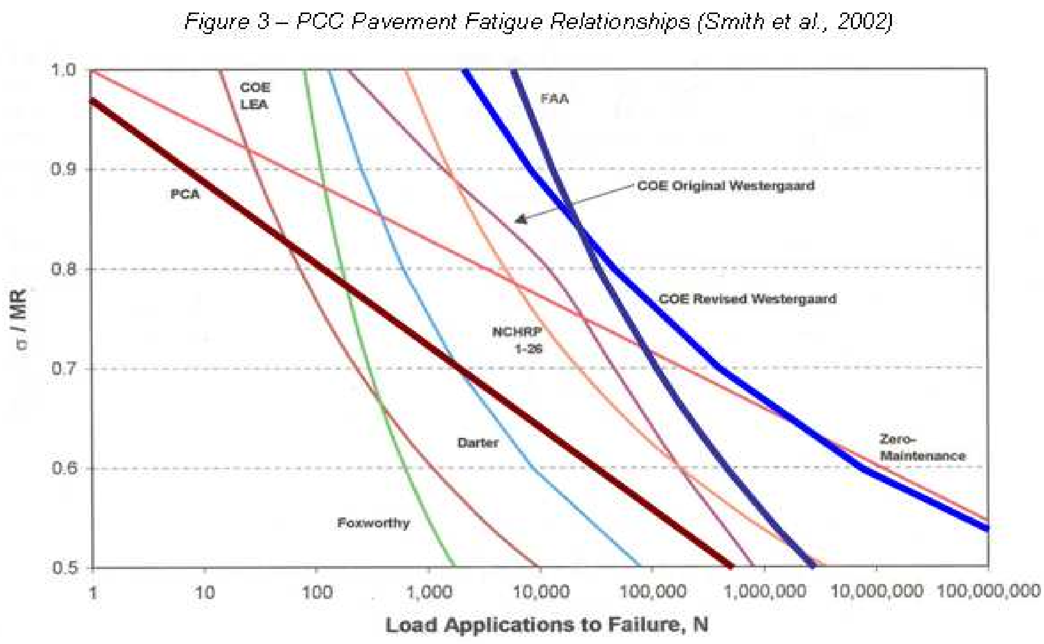


Figure 2 Existing Models for Airport Pavement Design [3][4]

THREE-STAGE FAILURE OBSERVED IN THE TESTS CONDUCTED AT FAA'S NAPTF

Pavement Strength Measured Under Static Step Loads

Full-scale static step-loads were applied at the free edge of concrete slabs to induce bottom-up and top-down cracks, and then to determine the pavement strength, [6]. Both top-down and bottom-up crack initializations were successfully recorded. Then the pavement strength was estimated using the collected test data with two assumptions: the pavement strength is similar at the slab top and bottom, and the residual stress at the slab top and bottom has the same magnitude but different signs. It has been found that the laboratory flexural strength of the cast beam was higher than the flexural strength of the beams saw-cut from the slabs. And the "pavement strength" estimated based on above two assumptions were in between the flexural strengths from the casted and cut beams. Detailed are presented in [6]. Therefore, Roesler et al's [8] [9] finding that "the slab flexural strength is 2.8 times on average higher than the beam flexural strength" does not agree with observations.

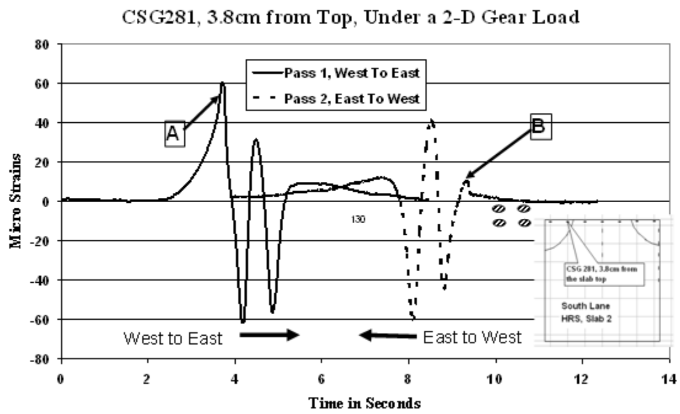
Pavement Strength Measured under Slow Rolling Loads

Figure 3 presents cracks and critical strains recorded in slab 2, South, test item HRS (Figure 4). Two corner cracks were observed in the slab after twenty-eight passes. No distress survey was conducted within the twenty-eight passes. Therefore how and when each crack was developed can only be found from the recorded strain histories.

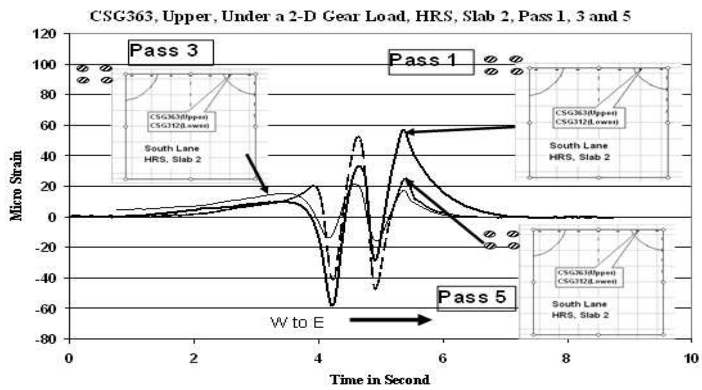
Gage 281 was located 122 cm (4 ft) from the west corner and 3.8 cm (1.5 inches) from the slab surface. The recorded maximum strain was about 60 micro strains (point A in Figure 3(a)). Since the slab thickness of test item HRS was 20.2 cm (9 inches) and the gage was installed 1.5 in from the surface for protection so the surface maximum strain estimated using thin plate theory is $60 \times (4.5/3) = 90$ micro strains. The Lab measured elastic modules of concrete was 40000 MPa (5800000 psi), so the measured maximum pavement surface stress (load induced portion) was about 3.6 MPa (522 psi) from the west to the east. When the same gear load moved from the east back to the west, the inverse strain reading of gage 281 was reduced to only 10 micro strains (point B in Figure 3(a)). The significant drop of stain indicated that crack was initiated by pass one. Or, the pavement strength (top-down, load induced portion) was about 3.6 MPa (522 psi).

The strain histories recorded by upper gage 363, 122 cm from the east corner of the same slab under pass 1, 3, and 5 are presented in Figure 3(b). It is clearly seen again that the crack was initiated after the pass one and the recorded 58 micro strains was close to the one recorded by gage 281. Therefore, the corresponding measured "pavement strength" (load related portion) was about 3.5 MPa (505 psi), close to that measured by gage 281.

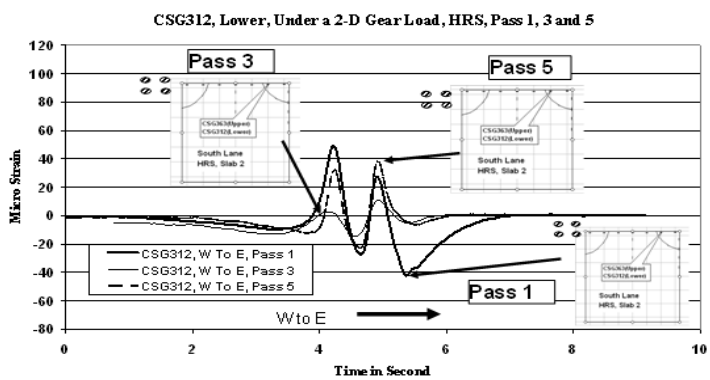
Figure 3(c) presents the strain histories of gage 312 that was at the location same to gage 363 but 3.8cm from the slab bottom. The histories under passes 1, 3, and 5 verify the reliability of the data in Figure 3(b): a top-down corner crack was developed by pass 1. The curves presented in Figure 3 show the detailed cracking process of the two corner cracks in south slab 2 in test item HRS. The slab was 22.9 cm thick on strong subgrade with CBR higher than 30. More and reliable data have also been received and available in FAA's database. Many cracks in the South slabs (Figure 4) were developed under one or two load application (pass 1 and/pass 2) and showed the strength failure rather than fatigue failure of the pavement.



(a) Left corner crack was developed after pass one and detected by gage 281



(b) Right corner crack was developed and detected by upper gage 363



(c) Right corner crack was also detected by lower gage 312

Figure 3 Cracks and Critical Strains Recorded in Slab 2, HRS.

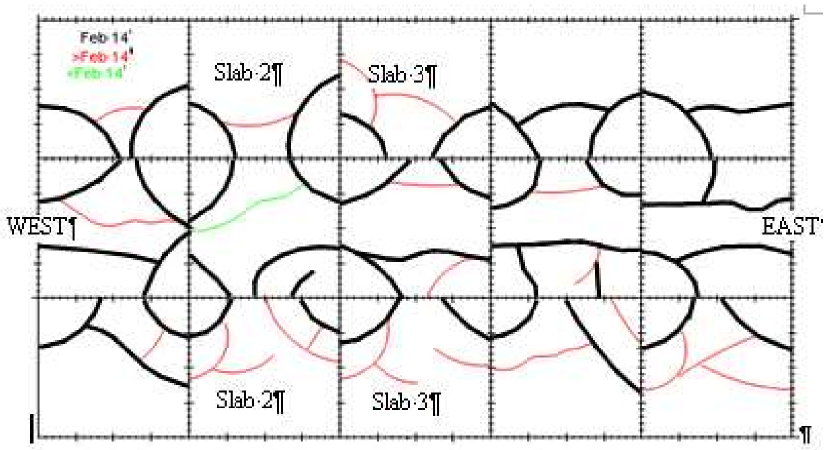


Figure 4 Failure Modes of HRS, CC1. Black cracks were observed after 28 passes on Feb. 14, 2000

The measured pavement strengths (load induced portion) under slow rolling loads were from 3.5 to 3.6 MPa (505 to 520 psi). After considering the tensile residual stress (0.5 – 1 MPa, 70 – 145 psi, [13]) existed near the slab surface before the load was moved on, the total stress initiating the crack would be about 4.0 to 4.6 MPa (580 to 670 psi). The average beam flexural strength obtained from Lab for HRS test pavement was 5.6 MPa (820 psi). It shows that the top-down pavement strength was approximately 18% to 28% lower than the beam flexural strength. The measured maximum strain in a concrete pavement should be always lower than the critical one since the strain gage can't be applied exactly at the critical stress location. Therefore, the difference between the beam flexural strength and the top-down pavement strength should be less than 28%. And the measured pavement top-down strengths were lower, not higher than the beam strength.

THREE-STAGE FAILURE OBSERVED IN THE TESTS PUBLISHED IN [8][9]

Significant efforts, including tests and analyses, were made by the concrete pavement team in University of Illinois since 1998 ([7][8][9]) to investigate the failure mechanism of concrete slabs. Review of their tests indicates that the tests were planned and conducted very well so reliable data have been obtained. The "three-stage" failure of a slab was also clearly observed from their data. However, the re-analysis of the data leads to the conclusion significantly different. The references [8] and [9] conclude: "… the slab's flexural strength was approximately 2.8 times higher than the beam flexural strength" (Abstract of [8], and page 1256, [9]). Our re-analysis of the same data still supports the assumption in FAA design: the pavement strength and simply supported beam flexural strength are relatively similar. What caused the different conclusions?

<u>The Three-Stage Failure Observed in Test Slab #1b, Figure 5</u>

The failure pattern of slab #1b tested by Illinois team is given in Figure 5 (Copied from [8], page 138). Load was applied at the slab edge. When the load magnitude was increased, the first crack was started at the edge then propagated to the slab center until the crack became "fully-hinged" [8][9] and shown in Figure 5. Monotonic loads were continuously applied until the second "Fully-hinged" crack was generated. Three-stage failure was clearly recorded in Figure 6.

Stage One E to A was a linear elastic zone in which the pavement structure was unique and constant. Linear elastic mechanistic model is valid for pavement response analysis in this zone - failure stage one.

Stage Two A to B1 was a zone with varying structure due to a varying crack depth and length. The linear elastic mechanistic model is not valid to analyze the slab response in this zone – failure stage two. The nonlinear relationship between load and deflection was clearly recorded and showed the slight decrease of slab stiffness due to the propagation of the first crack. B1 to B2 indicated the significant reduction of the slab stiffness after the first crack has been completely formed. B2 to B3 showed the behavior of second constant structure before the second crack was initiated.

Stage Three B3 to C1 was the zone with development of the second crack in Figure 5. The second varying structure is clearly shown with continuously decreased slab stiffness along with the propagation of the second crack. It seems that B3 indicated the initiation and C1 indicated the completion of the "fully hinged" second crack. C1 to C2 was still in the third failure stage.

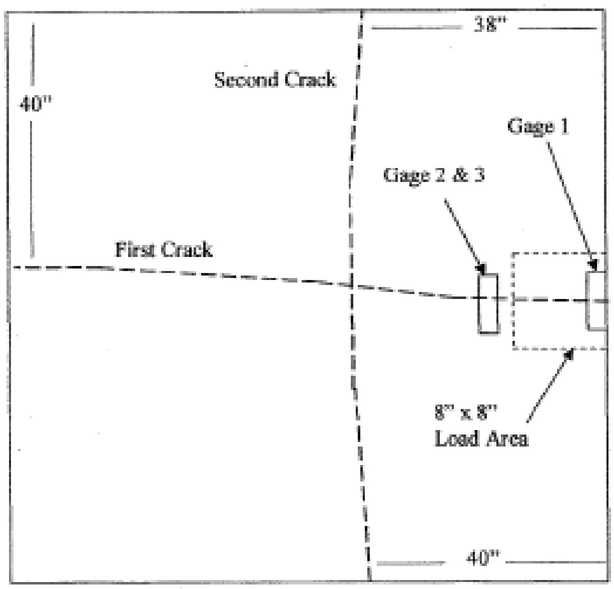


Figure 23 – Test #1b, Sketch of Slab #1 After Failure in Monotonic Loading

Figure 5 Copied from page 64, [8]

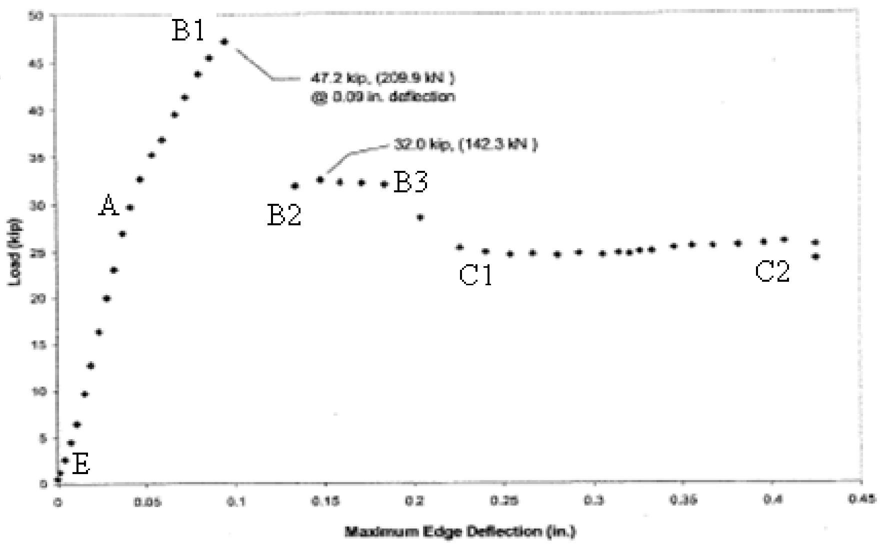


Figure 22 – Test #1b, Load versus Maximum Edge Deflection in Monotonic Loading

Figure 6 Copied from page 138, [8], 2004

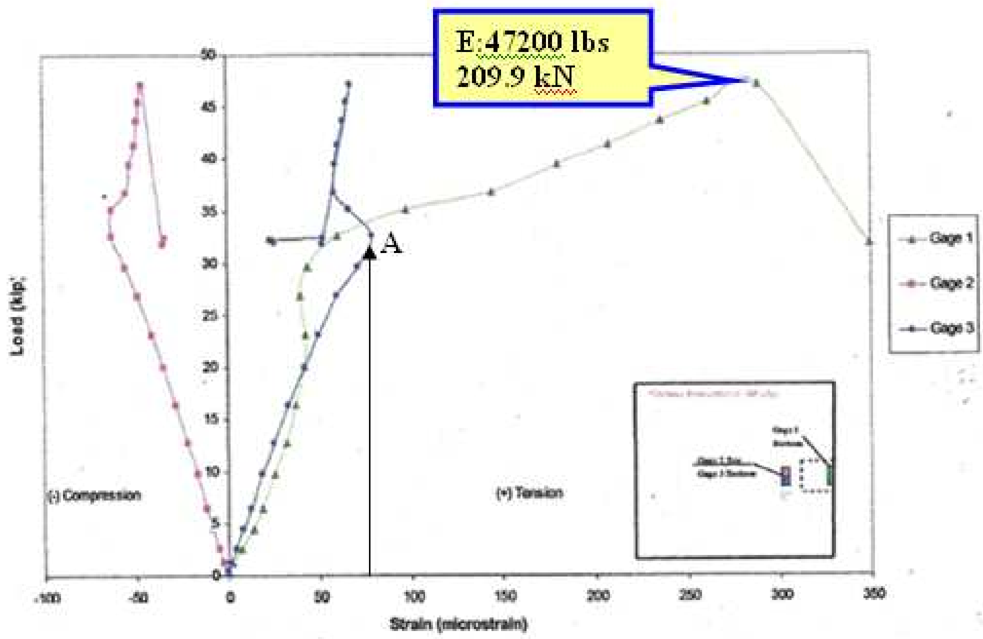


Figure 24 – Test #1b, Change of Strain with Monotonic Loading

Figure 7 Copied from page 139, [8], 2004

How Was the Slab Strength = 2.8 times of Beam Strength Calculated?

The results shown in Figure 6 were received under monotonic loads. Following statements were copied from page 79, [8] to show how the “slab flexural strength” was calculated: “Figure 22 (Figure 6 in this paper) plots the load versus maximum edge deflection. The first flexural crack occurred at 2143 kg (47.2-kips) and 2.3 mm (0.09 inch) deflection. This load level corresponded to a slab flexural strength (MOR_{Slab}) of 12.2 MPa (1770 psi) whereas companion beam specimens predicted a concrete flexural strength of 4.2 MPa (610 psi)”. Above statements indicate that the “Pavement strength” was calculated using Load = 2143 kg (47.2kips) and deflection 2.3 mm (0.09 inches) at the end of failure stage two defined in this paper. Or, it is point B1 in Figure 6 or point E in Figure 7.

12.2/4.2 = 2.9, that led to the conclusion “The concrete slabs’ flexural strength were on average 2.8 times of the strength of simply-supported beams”. Where 4.2 MPa (610 psi) was the beam flexural strength following ASTM C78 after considering the concrete age when the slab was tested (Figure 19, [8]).

How to calculate the “stress” in detail was described in following statements. “To determine the bending stress in the slab, each slab’s rebound deflection profile was matched with a finite element program ISLAB2000 (Khazanovich et al., - 2000) at 10 percent of the slab’s fatigue life (10% Nf).” “The maximum and minimum bending stress from ILLI-SLAB were recorded once the input deflection values were matched to actual test deflection values. Deflection values were matched by changing only the modulus of subgrade reaction (k-Value)” “… this method gives a reasonable estimate to the level of tensile bending stresses in the slab.” (page 101, [8] and page 1253, [9])

Above statements indicate that the “stress” (slab strength) at the end of failure stage two was determined by “changing only the modulus of subgrade reaction, k-Value” and “matching the calculated deflection equals to 2.3 mm (0.09 inches)”.

For verification purpose, above steps were followed using JSLAB2004 since ISLAB2000 is not available for us. The program ISLAB2000 and JSLAB2004 were evaluated by [10]. The authors concluded that the results calculated by using the two programs are identical for a single slab analysis. K(Final) was not given in [8] or [9]. It is found that the foundation modules k = 199 MPa/m (730 pci) leads to slab strength (critical stress at the edge) = 12.2 MPa (1770 psi) and deflection = 2.26 mm (0.089 inches) by JSLAB2004. All other input data are the same as presented in [8]. The results by ISLAB2000 (Copied from [8]) and calculated by JSLAB2000 are shown in Table 1. It has been verified that the procedure for verification in this paper was the same as used in [8] and [9] to obtain the conclusion: “the pavement strength is 2.8 times in average higher than the beam flexural strength.”

The stress (pavement strength) calculation procedure shown above for verification is only valid in failure stage one, up to point A in Figure 6 and Figure 7. However, the calculation was extended to the end of the second failure stage in [8] and [9]. After point A, the slab crack was initiated and the structure continuously varied so it should not be analyzed using a single slab – a structure without crack.

A simple and approximate estimation of the pavement strength (initiation of crack, load induced portion) can be obtained using the same data in Figures 6 and 7. To multiply 0.03/0.09 to 12.2 MPa (1770 psi), 4.07 MPa (590 psi) can be obtain, in which 0.03 was the deflection at the end of the first failure stage in Figure 6. 4.07 MPa (pavement strength) and 4.22 MPa (Beam flexural strength) do not have dramatic difference. It supports the findings in the full-scale tests conducted at the FAA's NAPTF and presented in previous sections.

The measured strains in Figure 7 can also be used to estimate the pavement strength as a double verification. The data of strain gages 2 and 3 are more reliable than those of gage 1 shown in Figure 7. Point A indicates the initiation of the first crack. The recorded strain was about 75 micro strains at the gage location embedded 1 inch from the slab bottom and the slab thickness was 20.3 cm (8 inches). So the maximum strain at the bottom was approximately $75 \times 4/3 = 100$ Micro strain. The elastic modulus E was about 3.5×10^6 psi (page 70, [8]). Therefore, the stress calculated based on the recorded strain by Gage 3 at the end of failure stage one would be about $100 \times 3.5 = 350$ psi (2.41 MPa) if the effects of poison's ratio is neglected ($\mu = 0$ is assumed). The stress was at 8 inches from the slab edge so the stress at the edge should be higher under the load in Figure 5. Using JSLAB2004, the ratio between the stress at the edge and at 8 inch from the edge is about 1.4. Therefore, the stress at the edge was about 350 *1.4 = 490 psi (3.4 MPa). Since the value of E was measured at the age 28 days and the test was conduced at the age 50 days. The value $E = 3.5 \times 10^6$ psi was under-estimated and the slab strength at age 50 days should be higher than 490 psi (3.4 MPa).

The slab strengths estimated using the deflection and strain recorded at the end of failure stage one in Figure 6 and 7 are listed in Table 2. They were lower, but in the same range with the beam flexural strength measured in the Lab (610 psi), rather than 2.8 times higher.

Table 1 Comparison of Results by Illinois and by Try-and-Error Method

	Calculated using Modules k = 199 MPa/m 730 psi, Load = 2143 kg (47.2 kps)	
	Def. (inches)	Critical Stress (psi)
ILLISLAB	0.09 [2]	1770 [2]
JSLAB	0.089	1780

Table 2 Estimated Slab Strength at the End of Failure Stage One in [8][9]

	Slab Strength
Using deflection data at 0.03 inches	590 psi
Using strain data at gage 3 (75 Micro Strain)	> 490 psi

COMMENTS ON THE FAILURE MODEL IN AC 150-5320/6C-6D (1978 – 1995)

The curve of FAA design model in AC 150-5320/6D [12] presented by [3] and [4] (Figure 2 in this paper) is not drawn correctly. The FAA model can be expressed as below:

$$\frac{\sigma}{MR} \leq \frac{1}{1.3 \times \alpha^2} \quad (1)$$

$$\begin{aligned} \alpha &= 1 + 0.15603 \times Log_{10}(\frac{COV}{5000}) \quad (COV \geq 5000) \\ \alpha &= 1 + 0.07058 \times Log_{10}(\frac{COV}{5000}) \quad (COV < 5000) \end{aligned} \quad (2)$$

COV = 5000 leads to $\alpha = 1$ (equation (2)) and $\sigma/MR \leq 1/1.3 = 0.77$ (equation (1)). Since the "FAA curve" in Figure 2 does not pass that point (N=5000, σ/MR=0.77) it can't be the true FAA curve.

COV is defined in AC150-5320 6C and 6D ([12], Appendix 2): For rigid pavements, coverage equal the number of times a pavement slab experiences a maximum stress application due to applied traffic. Therefore, COV is a parameter related to pavement service life to failure. MR is beam flexural strength of the concrete used to build the pavement, σ is the "maximum stress", defined above, at the edge of a slab and calculated using mechanistic model.

α is a factor for differentiating the failure behaviors of a concrete pavement under COV greater or smaller than 5000. $\alpha = 1$ for COV = 5000 is roughly related to the full-scale tests designed and conducted for developing the model.

A very important assumption embedded in equation (1) is: when σ = MR, the maximum stress in the pavement equals to the beam flexural strength of the concrete, the pavement can be used for COV = 5000 before it lost structural capability for providing accepted service. Or, when σ = MR, the pavement will fail in average after COV = 5000. For understanding the assumption, following concepts should be aware of:

(1) MR is strength of concrete as a property of material, not the strength of pavement as a property of structure;

(2) The structural "failure" of pavement (end of structural life) is corresponding to point C in Figure 1 (end of the third failure stage), neither point A nor point B. No any mechanistic model can be valid that far;

(3) At beginning, "1.3" had not been introduced in denominator in equation (1). When σ = MR, the full-scale tests found that the pavement survived approximately 5000 coverage in average. Therefore, based on statistics, only 50% of pavements will be serve for COV = 5000 or longer. For increasing the reliability of pavement life, safety factor 1.3 was introduced in equation (1). The probability of the pavement service life (expressed by COV) being longer than COV = 5000 should be significantly greater

than 50% after 1.3 was introduced in the model. However, we do not know what the probability is (70%? 80%? 90%?) since it can only be predicted using a probabilistic model. Equation (1) is a deterministic, empirical model, neither a probabilistic nor a mechanistic one.

(4) As discussed previously, σ is the load induced maximum stress calculated using the selected mechanistic model, and MR is the beam flexural strength. Many unknown factors still exist even if it is assumed that the mechanistic model can accurately predict the stress under any load. The "load induced stresses" are mostly lower than the total stress near the surface of slab and higher than the total stress near the slab bottom under a wheel load. It is not the "Critical" stress in a pavement anyway. MR is a material property obtained at end of failure stage one. Therefore, both σ and MR are parameters valid in failure stage one. However, the ratio between them is employed to predict pavement performance (or life) up to the end of failure stage three. Therefore, the accuracy of equation (1) is mainly governed by full-scale test results plus engineers' experiences such as the selected parameters 1.3, 0.15603, 0.07058, rather than by the accuracy or precision of the calculated load induced stress - σ. It must be emphasized again here that σ is not the critical stress in a pavement; rather, it is only partially responsible for cracking a pavement.

The FAA approach since 1978 defined as "fatigue analysis" is based on one assumption: the material strength MR obtained in Lab is relatively close to the concrete pavement strength in field. Therefore, though the stress and strength are obtained from two structures (beam for the flexural strength and pavement for the stress), the "fatigue analysis" concept is still approximately acceptable. If the pavement strength is significantly different from the beam strength, such as 2.8 times difference, the model should not be called "fatigue model" any more. All other models under the FAA after 6D, such as 6E and FARFIELD, were also based on the same assumption. This paper concludes that the FAA assumption since 1970's is still acceptable since it has been verified by the test data at the FAA's NAPTF and the model slab tests in University Illinois [8][9] if the data are analyzed in appropriate stage.

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REFERNECES

1 Rollings, R 1988. "Design of Overlays for Rigid Airport Pavements", Technical Report DOT/FAA/PM-87/19

2 Garg, N and E. Guo 2004, “Operational Life of Airport Pavements”, Technicla Report DOT/FAA/AR-04/46

3 Smith, K. D, J. R. Roesler, J. E. Naughton, 2002, “Review of Fatigue Models for Concrete Airfield Pavement Design”, Technical Report Prepared for American Concrete Pavement Association.

4 Thuma, Richard G. 2002: “A comparison of Rigid Airfield Pavement Design Methods”, Technical Report Prepared for American Concrete Pavement Association.

5 Miner, M.A., 1945, “Cumulative Damage In Fatigue”, Transactions, American Sociey of Mechanical Engineer, Vol. 67, pp159-164.

6 Guo, E.H, F. Petch and L. Ricalde, 2008 “Pavement Strength Measured by Full Scale Test”, Pavement Cracking, Mechanisms, Modeling, Detection, Testing and Case Histories, edited by Imad L. Al-Qadi, Tom Scarpas and Andreas Loizos, page 25-34, Published by CRC Press, Taylor & Francis Group.

7 Roesler, J. 1998 “Fatigue of Concrete Beams and Slabs”, Ph.D thesis of University fo Illinois, Urbana-Champaign.

8 Roesler, J, P. Littlton, J. Hiller and G. Long, 2004 “Effect of Stress State on Concrete Slab Fatigue Resistance” Technical Draft Final Report of Research Supported by the FAA under Grand DOT 95-C-001.

9 Roesler, J.R., Hiller, J.E., and Littleton, P.C. 2005 Large-Scale Airfield Concrete Slab Fatigue Tests, 9th International Conference on Concrete Pavement, pp 1247-1268, August 13-18, 2005, Colorado Springs

10 Wang, Weijun, Imad Basheer, and Katherine Petros, 2006 “Jointed Plain Concrete Pavement Model Evaluation”, Transportation research record, ISSN 0361-1981, TRB 85th Annual Meeting in January 2006

11 ASTM C78, “Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)”

12 FAA 1995, FAA Advisory Circular, Airport Pavement Design and Evaluation, AC150/5320 6D.

13 Guo, E, F. Pecht, L. Ricalde, D. Barbagallo and X. Li 2009, “Residual Stress Study at the FAA’s NAPTF**”,** 2nd European Airport Pavement Workshop, CROW, May 13-14, 2009, Netherlands.